EN 1998-6 (2005) (English): Eurocode 8: Design of structures for earthquake resistance – Part 6: Towers, masts and chimneys [Authority: The European Union Per Regulation 305/2011, Directive 98/34/EC, Directive 2004/18/EC]

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Eurocode 8: Design of structures for earthquake resistance - Part 6: Towers, masts and chimneys

Eurocode 8: Calcul des structures pour leur résistance aux séismes - Partie 6 : Tours, mâts et cheminées

Eurocode 8: Auslegung von Bauwerken gegen Erdbeben - Teil 6: Türme, Maste und Schornsteine

This European Standard was approved by CEN on 25 April 2005.

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EUROPÄISCHES KOMITEE FÜR NORMUNG

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Contents

FOREWORD

This European Standard EN 1998-6, Eurocode 8: Design of structures for earthquake resistance: Towers, masts and chimneys, has been prepared by Technical Committee CEN/TC 250 "Structural Eurocodes", the secretariat of which is held by BSI. CEN/TC 250 is responsible for all Structural Eurocodes.

This European Standard shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by December 2005 and conflicting national standards shall be withdrawn at latest by March 2010.

This document supersedes ENV 1998-3:1996.

According to the CEN-CENELEC Internal Regulations, the National Standard Organisations of the following countries are bound to implement this European Standard: Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

Background of the Eurocode programme

In 1975, the Commission of the European Community decided on an action programme in the field of construction, based on article 95 of the Treaty. The objective of the programme was the elimination of technical obstacles to trade and the harmonisation of technical specifications.

Within this action programme, the Commission took the initiative to establish a set of harmonised technical rules for the design of construction works which, in a first stage, would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them.

For fifteen years, the Commission, with the help of a Steering Committee with Representatives of Member States, conducted the development of the Eurocodes programme, which led to the first generation of European codes in the 1980s.

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement[1] between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to CEN through a series of Mandates, in order to provide them with a future status of European Standard (EN). This links *de facto* the Eurocodes with the provisions of all the Council's Directives and/or Commission's Decisions dealing with European standards (*e.g.* the Council Directive 89/106/EEC on construction products - CPD - and Council Directives 93/37/EEC, 92/50/EEC and 89/440/EEC on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market).

The Structural Eurocode programme comprises the following standards generally consisting of a number of Parts:

EN 1990 Eurocode: Basis of structural design

EN 1991 Eurocode 1: Actions on structures

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[1] Agreement between the Commission of the European Communities and the European Committee for Standardisation (CEN) concerning the work on EUROCODES for the design of building and civil engineering works (BC/CEN/03/89).

EN 1992 Eurocode 2: Design of concrete structures

EN 1993 Eurocode 3: Design of steel structures

EN 1994 Eurocode 4: Design of composite steel and concrete structures

EN 1995 Eurocode 5: Design of timber structures

EN 1996 Eurocode 6: Design of masonry structures

EN 1997 Eurocode 7: Geotechnical design

EN 1998 Eurocode 8: Design of structures for earthquake resistance

EN 1999 Eurocode 9: Design of aluminium structures

Eurocode standards recognise the responsibility of regulatory authorities in each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level where these continue to vary from State to State.

Status and field of application of Eurocodes

The Member States of the EU and EFTA recognise that Eurocodes serve as reference documents for the following purposes:

- as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 – Mechanical resistance and stability – and Essential Requirement N°2 – Safety in case of fire;
- as a basis for specifying contracts for construction works and related engineering services;
- as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs)

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents[2] referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards[3]. Therefore, technical aspects arising from the Eurocodes work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product standards with a view to achieving full compatibility of these technical specifications with the Eurocodes.

The Eurocode standards provide common structural design rules for everyday use for the design of whole structures and component products of both a traditional and an innovative nature. Unusual forms of construction or design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases.

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[2] According to Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for harmonised ENs and ETAGs/ETAs.

[3] According to Art. 12 of the CPD the interpretative documents shall:

a) give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;

b) indicate methods of correlating these classes or levels of requirement with the technical specifications, *e.g.* methods of calculation and of proof, technical rules for project design, etc. ;

c) serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.

The Eurocodes, *de facto*, play a similar role in the field of the ER 1 and a part of ER 2.

National Standards implementing Eurocodes

The National Standards implementing Eurocodes will comprise the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National title page and National foreword, and may be followed by a National annex.

The National annex may only contain information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, i.e:

- values and/or classes where alternatives are given in the Eurocode,
- values to be used where a symbol only is given in the Eurocode,
- country specific data (geographical, climatic, etc.), e.g. snow map,
- the procedure to be used where alternative procedures are given in the Eurocode.

It may also contain

- decisions on the use of informative annexes, and
- references to non-contradictory complementary information to assist the user to apply the Eurocode.

Links between Eurocodes and harmonised technical specifications (ENs and ETAs) for products

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works[4]. Furthermore, all the information accompanying the CE Marking of the construction products which refer to Eurocodes shall clearly mention which Nationally Determined Parameters have been taken into account.

Additional information specific to EN 1998-6

For the design of structures in seismic regions the provisions of this standard are to be applied in addition to the provisions of the other relevant Eurocodes. In particular, the provisions of the present standard complement those of Eurocode 3, Part 3-1 " Towers and Masts " and Part 3-2 " Chimneys", which do not cover the special requirements for seismic design.

National annex for EN 1998-6

Notes indicate where national choices have to be made. The National Standard implementing EN 1998-6 shall have a National annex containing values for all Nationally Determined Parameters to be used for the design in the country. National choice is required in the following sections.

[4] see Art.3.3 and Art.12 of the CPD, as well as clauses 4.2, 4.3.1, 4.3.2 and 5.2 of ID 1.

Reference section	Item
1.1(2)	Informative Annexes A, B, C, D, E and F.
3.1(1)	Conditions under which the rotational component of the ground motion should be taken into account.
3.5(2)	The lower bound factor β on design spectral values, if site-specific studies have been carried out with particular reference to the long-period content of the seismic action.
4.1(5)P	Importance factors for masts, towers, and chimneys.
4.3.2.1(2)	Detailed conditions, supplementing those in 4.3.2.1(2), for the lateral force method of analysis to be applied.
4.7.2(1)P	Partial factors for materials
4.9(4)	Reduction factor ν for displacements at damage limitation limit state

1 GENERAL

1.1 Scope

(1) The scope of Eurocode 8 is defined in EN 1998-1:2004, **1.1.1** and the scope of this Standard is defined in **(2)** to **(4)**. Additional parts of Eurocode 8 are indicated in EN 1998-1:2004, **1.1.3**.

(2) EN 1998-6 establishes requirements, criteria, and rules for the design of tall slender structures: towers, including bell-towers, intake towers, radio and TV-towers, masts, chimneys (including free-standing industrial chimneys) and lighthouses. Additional provisions specific to reinforced concrete and to steel chimneys are given in Sections **5** and **6**, respectively. Additional provisions specific to steel towers and to steel guyed masts are given in Sections **7** and **8**, respectively. Requirements are also given for non-structural elements, such as antennae, the liner material of chimneys and other equipment.

NOTE 1 Informative Annex A provides guidance and information for linear dynamic analysis accounting for rotational components of the ground motion.

NOTE 2 Informative Annex B provides information and guidance on modal damping in modal response spectrum analysis.

NOTE 3 Informative Annex C provides information on soil-structure interaction and guidance for accounting for it in linear dynamic analysis.

NOTE 4 Informative Annex D provides supplementary information and guidance on the number of degrees of freedom and the number of modes of vibration to be taken into account in the analysis.

NOTE 5 Informative Annex E gives information and guidance for the seismic design of Masonry chimneys.

NOTE 6 Informative Annex F gives supplementary information for the seismic performance and design of electrical transmission towers.

(3) The present provisions do not apply to cooling towers and offshore structures.

(4) For towers supporting tanks, EN 1998-4 applies.

1.2 Normative References

1.2.1 Use

(1)P This European Standard incorporates by dated or undated reference, provisions from other publications. These normative references are cited at the appropriate places in the text and the publications are listed hereafter. For dated references, subsequent amendments to or revisions of any of these publications apply to this European Standard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication referred to applies (including amendments).

1.2.2 General reference standards

(1) EN 1998-1:2004, **1.2.1** applies.

1.2.3 Additional reference standards for towers, masts and chimneys

(1) EN 1998-6 incorporates other normative references cited at the appropriate places in the text. They are listed below:

EN 1990 Basis of structural design – Annex A3: Application for towers and masts.

EN 1992-1-1 Design of concrete structures – General rules and rules for buildings

EN 1992-1-2 Design of concrete structures – Structural fire design

EN 1993-1-1 Design of steel structures – General rules and rules for buildings

EN 1993-1-2 Design of steel structures – Structural fire design

EN 1993-1-4 Design of steel structures – Stainless steel

EN 1993-1-5 Design of steel structures – Plated structural elements

EN 1993-1-6 Design of steel structures – Strength and stability of shell structures

EN 1993-1-8 Design of steel structures – Design of joints

EN 1993-1-10 Design of steel structures – Selection of material for fracture toughness and through thickness properties

EN 1993-1-11 Design of steel structures – Design of structures with tension components made of steel

EN 1993-3-1 Design of steel structures – Towers and masts

EN 1993-3-2 Design of steel structures – Chimneys

EN 1994-1-1 Design of composite steel and concrete structures – General rules and rules for buildings

EN 1994-1-2 Design of composite steel and concrete structures – Structural fire design

EN 1998-1 Design of structures for earthquake resistance – General rules, seismic actions and rules for buildings

EN 1998-5 Design of structures for earthquake resistance – Foundations, retaining structures and geotechnical aspects.

EN 1998-2 Design of structures for earthquake resistance – Bridges.

EN 13084-2 Free-standing chimneys – Concrete chimneys

EN 13084-7 Free-standing chimneys – Product specification of cylindrical steel fabrications for use in single-wall steel chimneys and steel liners.

1.3 Assumptions

(1)P The general assumptions of EN 1990:2002, **1.3** and EN 1998-1:2004, **1.3**(2)P, apply.

1.4 Distinction between principles and application rules

(1) EN 1990:2002, **1.4** applies.

1.5 Terms and definitions

1.5.1 General terms and definitions

(1) EN 1998-1:2004, **1.5.1** and **1.5.2** apply.

(2) The definitions in EN 1993-3-1, **1.5** and EN 1993-3-2, **1.5** apply.

1.5.2 Further terms and definitions used in EN 1998-6

angle tower

transmission tower used where the line changes direction by more than 3^{o} in plan. It supports the same kind of loads as the tangent tower

dead-end towers (also called anchor towers)

transmission tower able to support dead-end pulls from all the wires on one side, in addition to the vertical and transverse loads

tangent tower

transmission tower used where the cable line is straight or has an angle not exceeding 3^{o} in plan. It supports vertical loads, a transverse load from the angular pull of the wires, a longitudinal load due to unequal spans, and forces resulting from the wire-stringing operation, or a broken wire

telescope joint

joint between tubular elements without a flange, the internal diameter of one being equal to the external diameter of the other

transmission tower

tower used to support low or high voltage electrical transmission cables

trussed tower

tower in which the joints are not designed to resist the plastic moment of the connected elements

1.6 Symbols

1.6.1 General

(1) EN 1998-1:2004, **1.6.1** and **1.6.2** apply.

(2) For ease of use, further symbols, used in connection with the seismic design of towers, masts and chimneys, are defined in the text where they occur. However, in addition, the most frequently occurring symbols used in EN 1998-6 are listed and defined in **1.6.2**.

1.6.2 Further symbols used in EN1998-6

E_{eq} equivalent modulus of elasticity;

M_{i} effective modal mass for the i-th mode of vibration;

R^{θ} ratio between the maximum moment in the spring of an oscillator with rotation as its single-degree-of-freedom, and the rotational moment of inertia about the axis of rotation. The diagram of R^{θ} versus the natural period is the rotation response spectrum;

R^{θ}_{x}, R^{θ}_{y}, R^{θ}_{z} rotation response spectra around the x, y and z axes, in rad/s^2;

γ unit weight of the cable;

σ tensile stress in the cable;

$\bar{\xi}_j$ equivalent modal damping ratio of the *j*-th mode.

1.7 S.I. Units

(1)P EN 1998-1:2004, **1.7**(1)P applies.

(2) EN 1998-1:2004, **1.7**(2) applies.

2 PERFORMANCE REQUIREMENTS AND COMPLIANCE CRITERIA

2.1 Fundamental requirements

(1)P For the types of structures addressed by this Eurocode, the no-collapse requirement in EN 1998-1:2004, **2.1**(1)P applies, in order to protect the safety of people, nearby buildings and adjacent facilities.

(2)P For the types of structures addressed by this Eurocode the damage limitation requirement in EN 1998-1:2004, **2.1**(1)P applies, in order to maintain the continuity of the operation of plants, industries and communication systems, in the event of earthquakes.

(3)P The damage limitation requirement refers to a seismic action having a probability of exceedance higher than that of the design seismic action. The structure shall be designed and constructed to withstand this action without damage and limitation of use, the cost of damage being measured with respect to the effects on the supported equipment and from the limitation of use due to disruption of operation of the facility.

(4) In cases of low seismicity, as defined in EN 1998-1:2004, **2.2.1**(3) and **3.2.1**(4), the fundamental requirements may be satisfied by designing the structure for the seismic design situation as non-dissipative, taking no account of any hysteretic energy dissipation and neglecting the rules of the present Eurocode that specifically refer to energy dissipation capacity. In that case, the behaviour factor should not be taken greater than the value of 1,5 considered to account for overstrengths (see EN 1998-1:2004, **2.2.2**(2)).

2.2 Compliance criteria

2.2.1 Foundation

(1)P Foundation design shall conform to EN 1998-5.

2.2.2 Ultimate limit state

(1) EN 1998-1:2004, **2.2.2** applies.

2.2.3 Damage limitation state

(1) In the absence of any specific requirement of the owner, the rules specified in **4.9** apply, to ensure that damage considered unacceptable for this limit state will be prevented to the structure itself, to non-structural elements and to installed equipment. Deformation limits are established with reference to a seismic action having a probability of occurrence higher than that of the design seismic action, in accordance with EN 1998-1:2004, **2.1**(1)P.

(2) Unless special precautions are taken, provisions of this Eurocode do not specifically provide protection against damage to equipment and non-structural elements under the design seismic action, as this is defined in EN 1998-1:2004, **2.1**(1)P.

3 SEISMIC ACTION

3.1 Definition of the seismic input

(1) In addition to the translational components of the earthquake motion, defined in EN 1998-1:2004, **3.2.2** and **3.2.3**, the rotational component of the ground motion should be taken into account for tall structures in regions of high seismicity.

NOTE 1: The conditions under which the rotational component of the ground motion should be taken into account in a country, will be found in the National Annex. The recommended conditions are structures taller than 80 m in regions where the product a_gS exceeds 0,25g.

NOTE 2: Informative Annex A gives a possible method to_define the rotational components of the motion and provides guidance for taking them into account in the analysis.

3.2 Elastic response spectrum

(1)P The elastic response spectrum in terms of acceleration is defined in EN 1998-1:2004, **3.2.2.2** for the horizontal translational components and in EN 1998-1:2004, **3.2.2.3** for the vertical translational component.

3.3 Design response spectrum

(1) The design response spectrum is defined in EN 1998-1:2004, **3.2.2.5**. The value of the behaviour factor, q, reflects, in addition to the hysteretic dissipation capacity of the structure, the influence of the viscous damping being different from 5%, including damping due the soil-structure interaction (see EN 1998-1:2004, **2.2.2**(2), **3.2.2.5**(2) and (3)).

(2) For towers, masts and chimneys, depending on the cross section of the members, design for elastic behaviour until the Ultimate Limit State may be appropriate. In this case the q factor should not exceed $q = 1{,}5$.

(3) Alternatively to (2), design for elastic behaviour may be based on the elastic response spectrum with $q = 1{,}0$ and values of the damping which are chosen to be appropriate for the particular situation in accordance with **4.2.4**.

3.4 Time-history representation

(1) EN 1998-1:2004, **3.2.2.5** applies to the representation of the seismic action in terms of acceleration time-histories. In the case of the rotational components of the ground motion, rotational accelerations are simply used instead of translational ones.

(2) Independent time-histories should be used for any two different components of the ground motion (including the translational and the rotational components).

3.5 Long period components of the motion at a point

(1) Towers, masts and chimneys are often sensitive to the long-period content of the ground motion. Soft soils or peculiar topographic conditions might provide unusually large amplification of the long-period content of the ground motion. This amplification should be taken into account as appropriate.

NOTE: Guidance on the assessment of soil type for the purpose of determining appropriate ground spectra is given in EN 1998-5:2004, **4.2.2** and in EN 1998-1:2004, **3.1.2**. Guidance on cases where topographical amplification of motion may be significant is given in Informative Annex A of EN 1998-5:2004.

(2) Where site-specific studies have been carried out, with particular reference to the long period content of the motion, lower values of the factor β in expression (3.16) of EN 1998-1:2004 are appropriate.

NOTE: The value to be ascribed to β for use in a country, in those cases where site-specific studies have been carried out with particular reference to the long-period content of the motion, can be found in its National Annex. The recommended value for β in such a case is 0,1.

3.6 Ground motion components

(1) The two horizontal components and the vertical component of the seismic action should be taken as acting simultaneously.

(2) When taken into account, the rotational components of the ground motion should be taken as acting simultaneously with the translational components.

4 DESIGN OF EARTHQUAKE RESISTANT TOWERS, MASTS AND CHIMNEYS

4.1 Importance classes and importance factors

(1)P Towers, masts and chimneys are classified in four importance classes, depending on the consequences of collapse or damage, on their importance for public safety and civil protection in the immediate post-earthquake period, and on the social and economic consequences of collapse or damage.

(2) The definitions of the importance classes are given in Table 4.1.

Table 4.1 Importance classes for towers, masts and chimneys

Importance class	
I	Tower, mast or chimney of minor importance for public safety
II	Tower, mast or chimney not belonging in classes I, III or IV
III	Tower, mast or chimney whose collapse may affect surrounding buildings or areas likely to be crowded with people.
IV	Towers, masts or chimneys whose integrity is of vital importance to maintain operational civil protection services (water supply systems, an electrical power plants, telecommunications, hospitals).

(3) The importance factor γ_I = 1,0 is associated with a seismic event having the reference return period indicated in EN 1998-1:2004, **3.2.1**(3).

(4)P The value of γ_I for importance class II shall be, by definition, equal to 1,0.

(5)P The importance classes are characterised by different importance factors γ_I, as described in EN 1998-1:2004, **2.1**(3).

NOTE The values to be ascribed to γ_I for use in a country may be found in its National Annex. The values of γ_I may be different for the various seismic zones of the country, depending on the seismic hazard conditions and on public safety considerations (see Note to EN 1998-1:2004, **2.1**(4)). The recommended values of γ_I for importance classes I, III and IV are equal to 0,8, 1,2 and 1,4, respectively.

4.2 Modelling rules and assumptions

4.2.1 Number of degrees of freedom

(1) The mathematical model should:

- take into account the rotational and translational stiffness of the foundation;
- include sufficient degrees of freedom (and the associated masses) to determine the response of any significant structural element, equipment or appendage;
- include the stiffness of cables and guys;
- take into account the relative displacements of the supports of equipment or machinery (for example, the interaction between an insulating layer and the exterior tube in a chimney);

- take into account piping interactions, externally applied structural restraints, hydrodynamic loads (both mass and stiffness effects, as appropriate).

(2) Models of electric transmission lines should be representative of the entire line. As a minimum, at least three consecutive towers should be included in the model, so that the cable mass and stiffness is representative of the conditions for the central tower.

(3) Dynamic models of bell-towers should take into account the oscillation of bells, if the bell mass is significant with respect to that of the top of the bell-tower.

4.2.2 Masses

(1)P The discretisation of masses in the model shall be representative of the distribution of inertial effects of the seismic action. Where a coarse discretisation of translational masses is used, rotational inertias shall be assigned to the corresponding rotational degrees of freedom.

(2)P The masses shall include all permanent parts, fittings, flues, insulation, any dust or ash adhering to the surface, present and future coatings, liners (including any relevant short- or long-term effects of fluids or moisture on the density of liners) and equipment. The permanent value of the mass of structures or permanent parts, etc., the quasi-permanent value of the equipment mass and of ice or snow load, and the quasi-permanent value of the imposed load on platforms (accounting for maintenance and temporary equipment) shall be taken into account.

(3)P The combination coefficients ψ_{Ei} introduced in EN 1998-1:2004, **3.2.4**(2)P, expression (3.17), for the calculation of the inertial effects of the seismic action shall be taken as equal to the combination coefficients $\psi_{2\mathrm{i}}$ for the quasi-permanent value of variable action q_{i}, as given in EN 1990:2002, Annex A3.

(4)P The mass of cables and guys shall be included in the model.

(5) If the mass of the cable or guy is significant in relation to that of the tower or mast, the cable or guy should be modelled as a lumped mass system.

(6)P The total effective mass of the immersed part of intake towers shall be taken as equal to the sum of:

- the actual mass of the tower shaft (without allowance for buoyancy),
- the mass of the water possibly enclosed within the tower (hollow towers),
- the added mass of the externally entrained water.

NOTE: In the absence of rigorous analysis, the added mass of entrained water may be estimated according to Informative Annex F of EN 1998-2:2005.

4.2.3 Stiffness

(1) In concrete elements the stiffness properties should be evaluated taking into account the effect of cracking. If design is based on a value of the q factor greater than 1, with the corresponding design spectrum, these stiffness properties should correspond to incipient yielding and may be determined in accordance with EN 1998-1:2004, **4.3.1**(6) and (7). If design is based on a value of q =1 and the elastic response spectrum or a corresponding time-history representation of the ground motion, the stiffness of concrete elements should be calculated from the cracked cross-section properties that are consistent with the level of stress under the seismic action.

(2) The effect of the elevated temperature on the stiffness and strength of the steel or of reinforced concrete, in steel or concrete chimneys, respectively, should be taken into account.

(3) If a cable is modelled as a single spring for the entire cable, instead of a series of lumped masses connected through springs, the stiffness of the single spring should account for the sag of the cable. This may be done by using the following equivalent modulus of elasticity:

$$E_{\mathrm{eq}} = \frac{E_{\mathrm{c}}}{1 + \frac{(\gamma \ell)^2}{12\sigma^3} E_{\mathrm{c}}} \tag{4.1}$$

where:

E_{eq} is the equivalent modulus of elasticity,

γ is the unit weight of the cable, including the weight of any ice load on the cable in the seismic design situation,

σ is the tensile stress in the cable,

ℓ is the cable length,

E_{c} is the modulus of elasticity of the cable material.

(4) For strands consisting of wrapped ropes or wires, E_{c} is generally lower than the modulus of elasticity E in a single chord. In the absence of specific data from the manufacturer, the following reduction may be taken:

$$\frac{E_{\mathrm{c}}}{E} = \cos^3\beta \tag{4.2}$$

where β is the wrapping angle of the single chord.

(5) If the preload of the cable is such that the sag is negligible, or if the tower is shorter than 40 m, then the cable may be modelled as a linear spring.

NOTE: The mass of the cable should be fully accounted for in accordance with **4.2.2**(4)P.

4.2.4 Damping

(1) If the analysis is performed in accordance with **3.3**(3) on the basis of the elastic response spectrum of EN 1998-1:2004, **3.2.2.2**, viscous damping different from 5% may be used. In that case, a modal response spectrum analysis may be applied with damping ratio taken to be different in each mode of vibration.

NOTE: A modal response spectrum analysis procedure accounting for modal damping is given in Informative Annex B.

4.2.5 Soil-structure interaction

(1) For structures founded on soft soil deposits, EN 1998-1:2004, **4.3.1**(9)P applies for the effects of soil-structure interaction.

NOTE 1: Informative Annex C provides guidance for taking soil-structure interaction into account in the analysis.

NOTE 2: In tall structures, e.g. with height being greater than five times the maximum base dimension, the rocking compliance of the soil is important and may significantly increase the second order effects.

4.3 Methods of analysis

4.3.1 Applicable methods

(1) The seismic action effects and the effects of the other actions included in the seismic design situation may be determined on the basis of linear-elastic behaviour of the structure.

(2) EN 1998-1:2004, **4.3.3.1**(2)P, (3), (4) and (5) apply.

NOTE: The Note to EN 1998-1:2004, **4.3.3.1**(4) applies.

(3)P For the "rigid diaphragm" assumption to be applicable to steel towers, a horizontal bracing system capable of providing the required rigid diaphragm action, shall be provided.

(4)P For the "rigid diaphragm" assumption to be applicable to steel chimneys, horizontal stiffening rings shall be provided at close spacing.

(5) If the conditions for the applicability of the "rigid diaphragm" assumption are not met, a three-dimensional dynamic analysis should be performed, capable of capturing the distortion of the structure within horizontal planes.

4.3.2 Lateral force method

4.3.2.1 General

(1) This type of analysis is applicable to structures that meet both of the following two conditions

(a) The lateral stiffness and mass distribution are approximately symmetrical in plan with respect to two orthogonal horizontal axes, so that an independent model can be used along each one of these two orthogonal axes.

(b) The response is not significantly affected by contributions of higher modes of vibration.

(2) For condition (1)b) to be met, the fundamental period in each one of the two horizontal directions of (1)a) should satisfy EN 1998-1:2004: **4.3.3.2.1**(2)a. In addition, the lateral stiffness, the mass and the horizontal dimensions of the structure should remain constant or reduce gradually from the base to the top, without abrupt changes.

NOTE: The detailed or additional conditions for the lateral force method of analysis to be applied in a country may be found in its National Annex. The recommended additional conditions are: a total height, H, not greater than 60 m and an importance class I or II.

(3) If the relative motion between the supports of piping and equipment supported at different points is important for the verification of the piping or the equipment, a modal response spectrum analysis should be used, to take into account the contribution of higher modes to the magnitude of this relative motion.

NOTE: The lateral force method of analysis might underestimate the magnitude of the differential motion between different points of the structure.

4.3.2.2 Seismic forces

(1) The analysis for the determination of the effects of the seismic action is performed by applying horizontal forces F_i, $i = 1, 2.....n$ to the n lumped masses to which the structure has been discretised, including the masses of the foundation. The sum of these forces is equal to the base shear, taken as equal to:

$$F_t = S_d(T)\sum_{1}^{n} m_j \tag{4.3}$$

where:

$S_d(T)$ is the ordinate of the design response spectrum as defined in EN 1998-1:2004, **3.2.2.5**, for the fundamental period of vibration T in the horizontal direction of the lateral forces. If the period T is not evaluated as in EN 1998-1:2004, **4.3.3.2.2**(2), the spectral value $S_d(T_C)$ should be used in expression (4.3).

(2) The distribution of the horizontal forces F_i to the n lumped masses should be taken in accordance with EN 1998-1:2004, **4.3.3.2.3.**

NOTE: The lateral force method normally overestimates the seismic action effects in tapered towers where the mass distribution substantially decreases with elevation.

4.3.3 Modal response spectrum analysis

4.3.3.1 General

(1) This method of analysis may be applied to every structure, with the seismic action defined by a response spectrum.

4.3.3.2 Number of modes

(1)P EN 1998-1:2004, **4.3.3.3.1**(2)P applies.

(2) The requirements specified in (1)P may be deemed to be satisfied if the sum of the effective modal masses for the modes taken into account amounts to at least 90% of the total mass of the structure.

NOTE 1: Informative Annex D provides further information and guidance for the application of (2).

NOTE 2: The number of modes which is necessary for the calculation of seismic actions at the top of the structure is generally higher than what is sufficient for evaluating the overturning moment or the total shear at the base of the structure.

NOTE 3: Nearly axisymmetric structures normally have very closely spaced modes which deserve special consideration.

4.3.3.3 Combination of modes

(1) EN 1998-1:2004, **4.3.3.3.2**(1), (2) and (3)P apply for the combination of modal maximum responses.

4.4 Combinations of the effects of the components of the seismic action

(1) The effects of any rotational component of the ground motion about a horizontal direction may be combined with those of the translational component in the orthogonal horizontal direction through the square root of the sum of the squares rule (SRSS combination).

(2) The combination of the effects of the components of the seismic action should be accounted for in accordance with either one of the two alternative procedures specified in EN 1998-1:2004, **4.3.3.5.2**(4). For the application of the procedure in EN 1998-1:2004, **4.3.3.5.2**(4) based on expressions (4.20) to (4.22), any rotational components about a horizontal direction should first be combined with those of the translational component in the orthogonal horizontal direction in accordance with (1).

4.5 Combinations of the seismic action with other actions

(1) EN 1990:2002, **6.4.3.4** and EN 1998-1:2004, **3.2.4**(1)P and (4) apply for the combination of the seismic action with other actions in the seismic design situation.

4.6 Displacements

(1) EN 1998-1:2004, **4.3.4**(1)P and (3) apply for the calculation of the displacements induced by the design seismic action.

4.7 Safety verifications

4.7.1 Ultimate limit state

(1)P The no-collapse requirement (ultimate limit state) under the seismic design situation is considered to be fulfilled if the conditions specified in the following subclauses regarding resistance of elements and connections, ductility and stability are met.

4.7.2 Resistance condition of the structural elements

(1)P The following relation shall be satisfied for all structural elements, including connections:

$$R_{d} \geq E_{d} \tag{4.4}$$

where:

R_{d} is the design resistance of the element, calculated in accordance with the mechanical models and the rules specific to the material (in terms of the characteristic value of material properties, f_{k}, and partial factors γ_{M}),

E_{d} is the design value of the action effect due to the seismic design situation (see EN 1990:2002 **6.4.3.4**), including, if necessary, second order effects. (see **4.7.3**) and thermal effects (see **4.8**). Redistribution of bending moments is permitted in accordance with EN 1992-1-1:2004, EN 1993-1-1:2004 and EN 1994-1-1:2004.

NOTE: The values ascribed to the partial factors for steel, concrete, structural steel, masonry and other materials for use in a country can be found in the relevant National Annex to this standard. In EN 1998-1:2004 notes to subclauses **5.2.4**(3), **6.1.3**(1), **7.1.3**(1) and **9.6**(3) refer to the values of partial factors for steel, concrete, structural steel and masonry for the design of new buildings in different countries.

4.7.3 Second order effects

(1)P Second order effects shall be taken into account, unless the condition in (2) is fulfilled.

(2) Second order effects need not be taken into account if the following condition is fulfilled:

$$\delta M/M_{o}<0,10 \tag{4.5}$$

where

δM is the overturning moment due to second order effect (P-Δ) effect,

M_{o} is the first-order overturning moment.

4.7.4 Resistance of connections

(1)P For welded or bolted non-dissipative connections, the resistance shall be determined in accordance with EN 1993-1-1.

(2)P The resistance to be provided for welded or bolted dissipative connections shall be greater than the plastic resistance of the connected dissipative member based on the design yield stress of the material as defined in EN 1993-1-1, taking into account the overstrength factor (see EN 1998-1, **6.1.3**(2) and **6.2**).

(3) For requirements and properties for bolts and welding consumables, EN 1993-1-8:2004 applies.

(4) Non-dissipative connections of dissipative members made by means of full penetration butt welds are deemed to satisfy the overstrength criterion.

4.7.5 Stability

(1)P The overall stability of the structure in the seismic design situation shall be verified, taking into account the effect of piping interaction and of hydrodynamic loads, where relevant for the seismic design situation.

(2) The overall stability may be considered to be verified, if the rules relevant to stability verification in EN 1992-1-1, EN 1993-1-1, EN 1993-1-5, EN 1993-1-6, EN 1993-3-1 and EN 1993-3-2 are fulfilled.

(3) The use of class 4 sections is allowed in structural steel members, provided that all of the following conditions are met:

(a) the specific rules in EN 1993-1-1:2004, **5.5** are fulfilled;

(b) the value of the behaviour factor, q, is limited to 1,5 (see also special rules in Sections **6** or **7** for structures with class 4 sections); and

(c) the slenderness λ is not greater than:

– 120 in leg members;

– 180 in seismic primary bracing members;

– 250 in seismic secondary bracing members;

where seismic primary and seismic secondary members are defined as in EN 1998-1:2004, **4.2.2**.

4.7.6 Ductility and energy dissipation condition

(1)P The structural elements and the structure as a whole shall possess capacity for ductility and energy dissipation which is sufficient for the demands under the design seismic action. The value of the behaviour factor used in the design should be related to the ductility and energy dissipation capacity of the structure.

(2) The requirement in (1)P is deemed to be satisfied through either one of the following design approaches:

(a) Design the structure for dissipative behaviour, using a value of the behaviour factor greater than 1,5 and applying the special rules given in Sections **5**, **6**, **7** and **8** for energy dissipation capacity of the different types of structures addressed in those Sections.

(b) Design the structure for non- (or low-) dissipative behaviour, using a value of the behaviour factor not greater than 1,5 and applying **2.1**(4).

4.7.7 Foundations

(1)P EN 1998-1:2004, **2.2.2**(4)P applies.

(2) The design and verification of the foundation should be in accordance with EN 1998-1:2004, **4.4.2.6.** When the action effect from the analysis for the design seismic action, $E_{F,E}$, in expression (4.30) of EN 1998-1:2004 is the vertical force due to the earthquake, N_{Ed}, the contribution of the vertical component of the seismic action to N_{Ed} may be neglected if it causes uplift of the foundation.

4.7.8 Guys and fittings

(1) For requirements and properties of ropes, strands, wires and fittings, EN 1993-1-11 applies.

4.8 Thermal effects

(1) The thermal effects of the normal operating temperature on the mechanical properties of the structural elements, such as the elastic modulus and the yield stress, should be taken into account in accordance with EN 1992-1-2:2004, EN 1993-1-2:2004 and EN 1994-1-2:2004. Thermal effects of structural element temperatures less than 100°C may be neglected. For free-standing steel chimneys, see EN 13084-7.

4.9 Damage limitation state

(1) The damage limitation requirement establishes limits to displacements under the damage limitation seismic action. Sections **5**, **6**, **7** and **8** provide limits depending on the type of structure.

(2) If the operation of the structure is sensitive to deformations, (for example in telecommunication towers, where deformation might lead to permanent damage of equipment or loss of the signal), reduced limits to displacements may be used.

(3) Displacements for the damage limitation requirement may be calculated as those obtained in accordance with **4.6**(1) for the design seismic action corresponding to the "ultimate limit state requirement" multiplied by a reduction factor ν which takes into account the lower return period of the seismic action associated with the damage limitation requirement (see EN 1998-1:2004, **4.4.3.1**).

(4) The value of the reduction factor ν may also depend on the importance class of the structure.

NOTE The values to be ascribed to ν for use in a country may be found in its National Annex. Different values of ν may be defined for the various seismic zones of a country, depending on the seismic hazard conditions and on the damage limitation objectives, which may be different for towers, masts or chimneys. The recommended values of ν are $\nu = 0{,}4$ for importance classes III and IV and $\nu = 0{,}5$ for importance classes I and II.

4.10 Behaviour factor

4.10.1 General

(1)P The value of the behaviour factor q shall be determined as:

$$q = q_0 k_r \geq 1{,}5 \tag{4.6}$$

where:

q_0 is the basic value of the behaviour factor, reflecting the ductility of the lateral load resisting system, with values defined in Sections **5**, **6**, **7** and **8** for each different type of structure,

k_r is the modification factor reflecting departure from a regular distribution of mass, stiffness or strength, with values defined in **4.10.2**.

4.10.2 Values of modification factor k_r

(1)P The value of k_r shall be taken as equal to 1,0, unless modified due to the existence of any of the following irregularities in the structure.

a) Horizontal eccentricity of the mass at a horizontal level with respect to the centroid of the stiffness of the elements at that level, exceeding 5% of the parallel dimension of the structure:

$$k_r = 0{,}8$$

b) Openings in a shaft or structural shell causing a 30% or larger reduction of the moment of inertia of the cross-section:

$$k_r = 0{,}8$$

c) Concentrated mass within the top third of the height of the structure, contributing by 50% or more to the overturning moment at the base:

$$k_r = 0{,}7$$

(2)P When more than one of the above irregularities are present, k_r shall be assumed to be equal to the product of 0,9 times the lowest values of k_r.

5 SPECIFIC RULES FOR REINFORCED CONCRETE CHIMNEYS

5.1 Scope

(1)P This section refers to concrete chimneys of annular (hollow circular) cross-section.

(2)P Concrete chimneys designed in accordance with this Eurocode shall conform to EN 1992-1-1:2004 and EN 1992-1-2:2004 and to the additional rules specified in this Section. For free-standing concrete chimneys, the rules of EN 13084-2:2001 that are complementary and non-contradictory to the rules of any EN-Eurocode apply also

(3) Concrete should be of a class not lower than C20/25, as defined in EN 1992-1-1:2004.

5.2 Design for dissipative behaviour

(1) Concrete chimneys may be designed for dissipative behaviour with a basic value of the behaviour factor q_o = 2,5, by applying within the critical sections defined in (2) the rules of the present clause **5.2**.

(2) The critical region should be taken as the following:

- from the base of the chimney to a height D above the base;
- from an abrupt change of section to a height D above the abrupt change of section;
- a height D above and below sections of chimney where more than one opening exists

where D is the outer diameter of the chimney at the middle of the critical region.

(3) In the design for dissipative behaviour, a minimum value of the local curvature ductility factor, μ_φ, should be provided within the critical sections defined in (2). The local curvature ductility factor should be ensured by providing confining reinforcement, in accordance with (4) and with EN 1998-1:2004, **5.4.3.2.2**(10)P and (11).

(4) The mechanical volumetric ratio of confining reinforcement, ω_{wd}, defined as in EN 1998-1:2004, **5.4.3.2.2**(8), should be related to the local curvature ductility factor, μ_ϕ, after spalling of the cover concrete, through the general method based on:

a) the definition of the curvature ductility factor from the curvatures at ultimate and at yielding, as $\mu_\phi=\phi_u/\phi_y$;

b) calculation of ϕ_u as $\phi_u=\varepsilon_{cu2,c}/x_u$ and of ϕ_y as $\phi_y=1,5f_y/(E_sD)$, where D is the diameter as defined in (2);

c) neutral axis depth, x_u, estimated from section equilibrium at ultimate conditions;

d) the stress-strain models in EN 1992-1-1:2004, **3.1.9** and the strength and ultimate strain of confined concrete, $f_{ck,c}$ and $\varepsilon_{cu2,c}$ as a function of the effective lateral confining stress in accordance with EN 1992-1-1:2004, **3.1.9**; and

e) expression of the effective lateral confining stress as $0,5\alpha\omega_{wd}$, with the confinement effectiveness factor α taken from EN 1998-1:2004, **5.4.3.2.2**(8)b) or c).

(5) The value of the curvature ductility factor, μ_ϕ, to be used in (3), (4) may be determined from the displacement ductility factor, μ_δ, using the expression:

$$\mu_\phi = \frac{\phi_u}{\phi_y} = 1 + \frac{\mu_\delta - 1}{4\frac{L_{pl}}{L_V}\left(1 - 0{,}5\frac{L_{pl}}{L_V}\right)} \tag{5.1}$$

where:

L_{pl}: plastic hinge length,

$L_V = M_{Ed}/V_{Ed}$: shear span of the chimney at the bottom section of the critical region calculated on the basis of the moment and shear from the analysis.

(6) The value of the displacement ductility factor, μ_δ, to be used in expression (5.1) may be derived from the following relationship between μ_δ and q_o:

$$\mu_\delta = q_o \qquad \text{if } T_1 \geq T_C \tag{5.2}$$

$$\mu_\delta = 1 + (q_o - 1)T_C/T_1 \qquad \text{if } T_1 < T_C \tag{5.3}$$

where T_1 is the fundamental period of the chimney T_C is the period at the upper limit of the constant acceleration region of the spectrum, in accordance with EN 1998-1:2004, **3.2.2.2**(2)P.

(7) The value of the plastic hinge length, L_{pl}, to be used in expression (5.1), may be taken equal to:

$$L_{pl} = 0{,}5D \tag{5.4}$$

where D is the outside diameter of the chimney as defined in (2).

(8) To avoid implosive spalling of the concrete at the inner surface, within the critical sections defined in (2) the value of the ratio of the outer diameter, as defined in (2), to the thickness of the section wall, should not exceed 20.

(9) Horizontal construction joints within the critical sections defined in (2) should be avoided.

(10) EN 1998-2:2005, **6.2.3** applies within the critical regions defined in (2).

5.3 Detailing of the reinforcement

5.3.1 Minimum reinforcement (vertical and horizontal)

(1)P In chimneys with an outer diameter, D, of 4 m or more, the vertical and the horizontal reinforcement shall be placed in two layers (curtains) each: one layer per direction near the inner and the other layer near the outer surface, with not less than half of the total vertical reinforcement placed in the layer near the outer face.

(2) In chimneys with an outer diameter of 4 m or more, the minimum ratio of the vertical reinforcement to the cross-sectional area should be not less than 0,003.

(3) In chimneys with an outer diameter of 4 m or more, the minimum ratio of the horizontal reinforcement to the cross-sectional area should be not less than 0,0025. For free-standing concrete chimneys, the relevant rule of EN 13084-2:2001 applies also.

(4)P In chimneys with an outer diameter of less than 4 m, the entire vertical or horizontal reinforcement may be placed in a single layer (curtain) per direction, near the outer surface. In that case the ratio of the reinforcement in the outer layer to the cross-sectional area should be not less than 0,002 per direction.

(5) Close to the chimney top, where stresses due to the permanent loads are low, the minimum vertical reinforcement ratio may be taken as equal to that of the horizontal reinforcement.

(6) The spacing of vertical bars should be not more than 250 mm and that of horizontal bars should be not more than 200 mm.

(7) The horizontal reinforcement bars should be placed between the vertical bars and the concrete surface. Cross-ties between the outer and the inner layer of reinforcement should be provided at a horizontal and vertical spacing of not more than 600 mm.

5.3.2 Minimum reinforcement around openings

(1) Around the perimeter and the corners of openings, reinforcement should be placed additional to that provided away from the openings. The additional reinforcement should include diagonal as well as vertical and horizontal bars at the corners and should be placed as near to the outside surface of the opening as normal constructional considerations permit. The bars should extend past the opening perimeter for a full anchorage length.

(2) The area of the additional horizontal and vertical reinforcement in each direction should not be less than that of the bars which are discontinued due to the presence of the opening. Over a horizontal distance from either vertical side of the opening of half the opening width, the vertical reinforcement ratio should not be less than 0,0075.

5.4 Special rules for analysis and design

(1) Except as specified in (2)P, only one horizontal component of the ground motion needs to be taken into account.

(2) P In chimneys with openings within the critical regions defined in **5.2**(2) with horizontal size greater than the thickness of the chimney wall, both horizontal components of the ground motion need to be taken into account.

(3) The vertical component of the ground motion may be disregarded.

(4) When the liner (consisting of brick, steel, or other materials) is laterally supported by the chimney structural shell at closely spaced points such that the movement of the liner relative to the shell is considered negligible, the mass of the liner may be incorporated into that of the structural shell, without including separate degrees of freedom for the liner.

(5) When the supports of the chimney liner at the top of the chimney and possibly at intermediate points permit movement of the liner relative to the structural shell, the liner should be included in the dynamic analysis model separately from the concrete structural shell. In that case, if the elastic response spectrum is used for the analysis in accordance with **3.3**(2) and **4.2.4**, the value of the damping ratio to be used for the liner should depend on its construction.

NOTE: Informative Annex B proposes values of the damping ratio for typical liner materials.

5.5 Damage limitation state

(1) Waste gas flues in chimneys should be checked for imposed deformations between support points and clearances between internal elements, so that gas tightness is not lost and sufficient reserve is maintained against collapse of the flue gas tube, under the displacements calculated in accordance with **4.9**(3).

(2) The requirement for damage limitation is considered to be satisfied if the lateral displacement of the top of the structure, calculated in accordance with **4.9**(3), does not exceed 0,5% of the height of the structure,

(3) The relative deflection between different points of support of the liner, computed in accordance with **4.9**(3), should be restricted for damage limitation of the liner. Unless stricter limits are specified for the particular project, the following limits on the relative lateral displacements of adjacent points of support of the liner should be observed:

a) if provisions are taken to allow relative movement between separate parts of the liner, (e.g. by constructing the liner of tubes independent from each other, with suitable clearance):

$$d_{\mathrm{r}} \leq 0{,}020\ \Delta H \tag{5.5}$$

b) in all other cases:

$$d_{\mathrm{r}} \leq 0{,}012\ \Delta H \tag{5.6}$$

where ΔH is the vertical distance of adjacent platforms supporting the liner.

6 SPECIAL RULES FOR STEEL CHIMNEYS

6.1 Design for dissipative behaviour

(1) Steel frame or truss structures which provide lateral support to flue gas ducts of chimneys may be designed for dissipative behaviour, in accordance with the relevant rules of EN 1998-1:2004, Section **6**. In that case their design should be based on values of the basic behaviour factor q_o not exceeding the following:

(a) moment resisting frames or frames with eccentric bracing $q_o = 5$;

(b) frames with concentric bracing: q_o taken from Fig. 7.1.

(2) Steel chimneys consisting of a structural shell designed for dissipative behaviour should satisfy the requirements of EN 1993-1-1:2004, **5.4.3** and **5.6** for plastic global analysis. In that case their design may be based on a value of the basic behaviour factor: $q_o = 2{,}5$.

(3) Depending on the chosen cross-sections, the basic value of the behaviour factor is limited by the values given in Table 6.1.

NOTE: Guyed steel chimneys are generally lightweight. As such, their design for lateral actions is usually governed by wind, unless they have large flares or other masses near the top.

Table 6.1: Restrictions on the basic value of the behaviour factor, depending on the cross-sectional class of steel elements

Basic value of the behaviour factor, q_o	Allowed cross-sectional class
$q_o \le 1{,}5$	Class 1, 2, 3 or 4 (in accordance with **4.7.5**(3))
$1{,}5 < q_o \le 2$	Class 1, 2 or 3
$2 < q_o \le 4$	Class 1 or 2
$q_o > 4$	Class 1

6.2 Materials

6.2.1 General

(1)P Structural steel shall conform to the European Standards referred to in EN 1993-1-1:2004, **1.2.2** and EN 1993-3-2.

(2)P Structural steel shall conform to EN 1993-1-1:2004, **3.2**

(3) The thickness of steel elements should conform to the requirements of EN 1993-1-10:2004, Table 2.1, depending on the Charpy V-Notch (CVN) energy and other relevant parameters, and of EN 1993-3-2.

(4) Where stainless steel or alloy steel components are connected to carbon steel, bolted connections are preferred. In order to avoid accelerated corrosion due to galvanic action, such connections should include insulating gaskets. Welding is permitted,

provided that specialised metallurgical control is exercised with regard to the welding procedure and the electrode selection.

6.2.2 Mechanical properties for structural carbon steels

(1)P The mechanical properties of structural carbon steels S 235, S 275, S 355, S 420, S 460 shall be taken from EN 1993-1-1:2004 and, for properties at higher temperatures, from EN 13084-7.

6.2.3 Mechanical properties of stainless steels

(1)P Mechanical properties related to stainless steels shall be taken from EN 1993-1-4 for temperature up to 400°C and at higher temperatures from EN 13084-7.

6.2.4 Connections

(1) For connection materials, welding consumables, etc., reference should be made to EN 1993-1-8:2004 and the relevant product standards specified therein.

NOTE: Reference is also made to EN 1993-3-2:2005, Informative Annexes C and E .

6.3 Damage limitation state

(1) **5.5(1)** applies.

(2) **5.5(2)** applies.

6.4 Ultimate limit state

(1) Design in accordance with the present standard, including the values of the behaviour factors specified for dissipative or for non-dissipative behaviour, is deemed to ensure that low cycle fatigue of structural details (especially connections) will not contribute to the ultimate limit state.

(2) In the design of details, such as flanges, the plastic stress distribution should be taken into account.

(3) In the verification of a chimney for the seismic design situation, a corrosion allowance on thickness should be taken into account in accordance with EN 1993-3-2, unless the special measures for corrosion protection in EN 1993-1-1:2004 are taken.

(4) Weakening of cross-section by cut-outs or openings (manholes, flue inlet) shall be compensated for by local reinforcement of the structural shell (e.g. through stiffeners around the edges of the openings), taking into account local stability considerations (see EN 1993-3-2).

7 SPECIAL RULES FOR STEEL TOWERS

7.1 Scope

(1)P Steel towers designed according to this Eurocode shall conform to the relevant parts of EN 1993, including EN 1993-1-1 and EN 1993-3-1, and to the additional rules specified in this Section.

7.2 Design for dissipative behaviour

(1) Design of steel towers for dissipative behaviour should be in accordance with the relevant rules of EN 1998-1:2004, Section **6.** In that case their design should be based on values of the basic behaviour factor q_o not exceeding the following:

(a) moment resisting frames, or frames with eccentric bracings $q_o = 5$;

(b) frames with concentric bracings: q_o taken from Fig. 7.1.

(2) **6.1**(3) applies.

(3) If trussed tubes are used in the major diagonals of the tower, the basic value of the behaviour factor should be limited to 2.

7.3 Materials

(1)P Structural steel shall conform to the European Standards referred to in EN 1993-1-1:2004, **1.2.2** and EN 1993-3-1.

(2)P **6.2.1**(2)P applies.

(3)P **6.2.1**(3)P applies.

(4) The requirements in EN 1998-1:2004, **6.2** apply.

(5) The thickness of cold-formed members for towers should be at least 3 mm.

NOTE: Steel towers are sometimes designed to be in service without maintenance for 30 years to 40 years or even longer. Weathering steel may then be used, unless protection against corrosion is applied, such as hot dip galvanising.

7.4 Design of towers with concentric bracings

(1) Figure 7.1 shows the values of q_o to be used in the design of typical configurations of steel towers with concentric bracings for dissipative behaviour.

(2) In the frames in Figure 7.1 (a) to (e) and (h), both the tension and compression diagonals shall be taken into account in an elastic analysis of the structure for the seismic action.

(3) The frames in Figure 7.1 (a) to (c) belong to K types of bracings and are not allowed for dissipative behaviour. The value of q for this type of frames is limited to 1,5.

(4) The frames in Figure 7.1(d) and (h) may be considered similar to V-braced frames with diagonals intersecting on a continuous horizontal member. Design for

dissipative behaviour should be in accordance with the rules given in EN 1998-1:2004, **6.7** pertaining to frames with V bracings.

(5) For the frame in Figure 7.1(e) design for dissipative behaviour should be in accordance with the rules in EN 1998-1:2004, **6.7** pertaining to frames with diagonal bracings in which the diagonals are not positioned as X diagonal bracings.

(6) The X-braced frames in Figure 7.1 (f) and (g) may be considered as frames with X diagonal bracings. In design for dissipative behaviour only the tension diagonals should be taken into account in an elastic analysis of the structure for the seismic action. Such design should be in accordance with the rules given in EN 1998-1:2004, **6.7** pertaining to frames with X diagonal bracings.

(7) If the value of the basic behaviour factor used in the design is greater than or equal to 3,5, fully triangulated horizontal bracings, such as those in Figure 7.2, should be provided.

7.5 Special rules for the design of electrical transmission towers

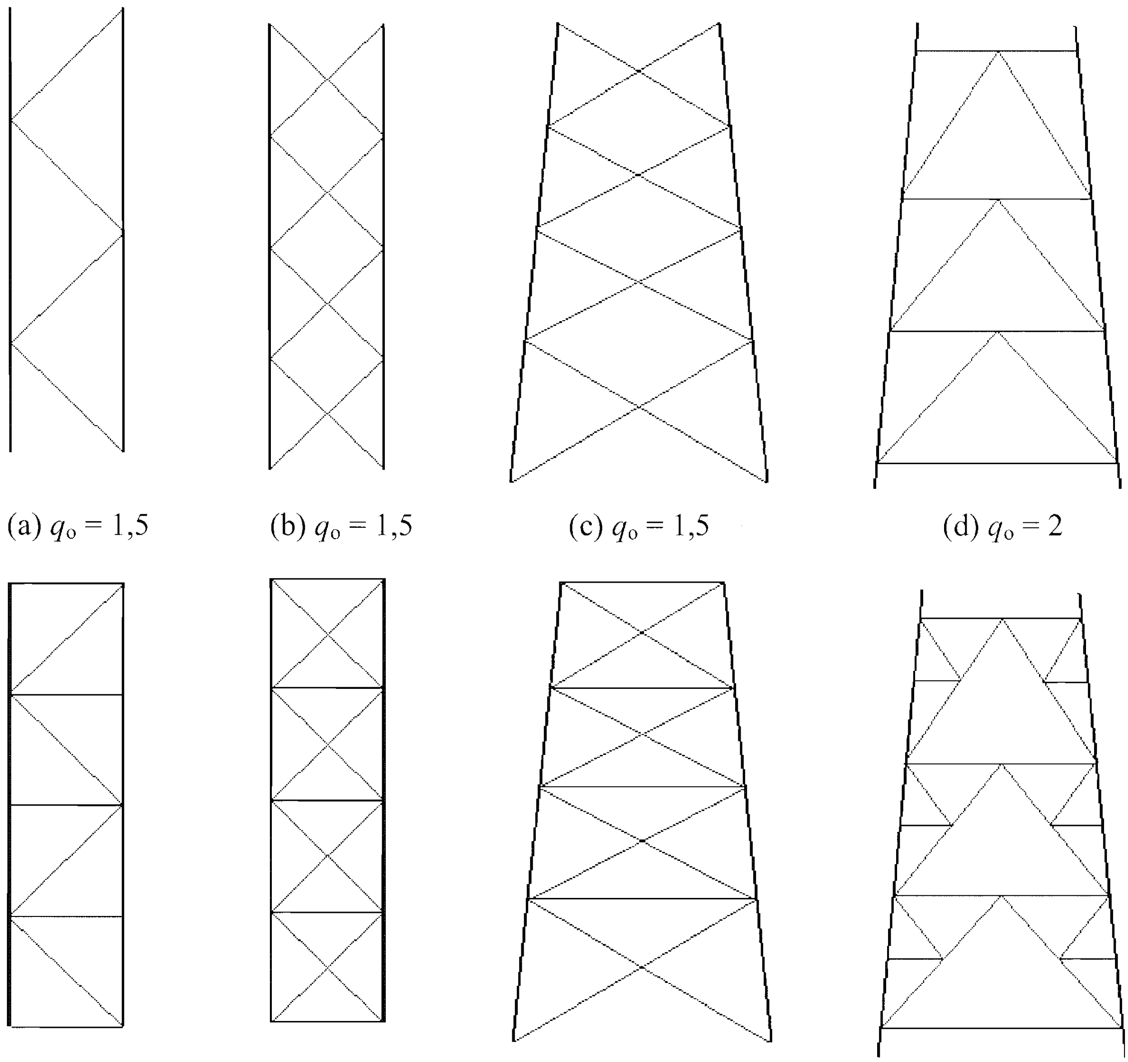
(1) The design should take into account the adverse effects on the tower of the cables between adjacent towers.

(2) The requirement in (1) may be satisfied if the seismic action effects in the tower structure are calculated by a simple addition of the following (SRSS or similar combination rules should not be used):

- The seismic action effects due to the forces exerted on the tower by the cables, assuming that the tower moves statically with respect to the adjacent ones in the most adverse direction. The assumed relative displacement should be equal to twice the design ground displacement specified in EN 1998-1:2004, **3.2.2.4**. A set of all physically possible relative displacements between towers should be analysed, under the assumption that towers are fixed at their base;
- The seismic action effects due to the inertia loads from a dynamic analysis in accordance with **4.2.1**(2). In the three towers model, a limiting assumption may be made for the two adjacent towers, if these are tangent towers. In this case, inertia loads may be calculated assuming the adjacent tower is elastically supported at the cable elevation along the direction of the cables.

7.6 Damage limitation state

(1) Limits on the displacements, calculated in accordance with **4.9**(3), should be specified for the particular project for the damage limitation state, depending on the function of the tower.



(e) $q_o = 3$ (f) $q_o = 4$ (g) $q_o = 4$ (h) $q_o = 2$

Figure 7.1: Basic values of the behaviour factor for configurations of steel frames with concentric bracings.

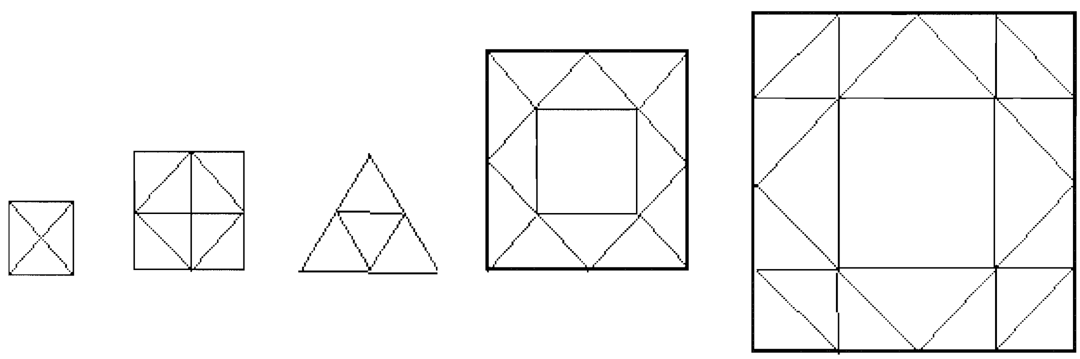


Figure 7.2: Examples of fully triangulated horizontal bracings, to be used in towers with $q_o \geq 3{,}5$.

7.7 Other special design rules

(1) "Telescope joints" may only be used in tubular steel towers, if they are experimentally qualified.

(2) Anchorage to the foundation should be provided at the base of the columns for the tension force which is the larger of the following two values, if they are tensile:

(a) the force calculated in accordance with **4.2.1**(2);

(b) the force calculated from the analysis for the seismic design situation, using a value of the behaviour factor not greater than $q = 2$.

(3) Joints in towers should be designed and detailed to meet the relevant requirements in EN 1998-1:2004, Section **6** for joints in structural systems of similar type and configuration, designed for the same basic value of the behaviour factor, q_o, as the tower.

8 SPECIAL RULES FOR GUYED MASTS

8.1 Scope

(1)P This section refers to steel masts.

(2)P Steel masts designed according to this Eurocode shall conform to the relevant parts of EN 1993, including EN 1993-1-1 and EN 1993-3-1, and to the additional rules specified in this Section.

8.2 Special analysis and design requirements

(1) Design for dissipative behaviour is not allowed in guyed masts. They should be designed for low dissipative behaviour with q = 1,5.

(2)P The stress in the guy cables due to the design seismic action shall be lower than the preload stress of the cable.

(3) The elastic restraint provided by the guy cables to the mast should be taken into account as follows:

– in relatively short masts (up to 30 or 40m) the guy cables may be considered to act as simple tension ties, with stiffness that remains constant as the mast bends;

– in taller towers the sag of the guy cables is large and should be accounted for through a cable stiffness that depends on deformations in accordance with **4.2.3**(2) and (3).

(4) The sagging of guy cables due to the ice load considered in the seismic design situation should be taken into account.

(5) For both sagging and straight cables, the horizontal component of the guy cable stiffness should be taken equal to:

$$\mathrm{K}_{\mathrm{eff,h}} = \cos^2\alpha \frac{A_{\mathrm{c}} E_{\mathrm{eq}}}{\ell} \tag{8.1}$$

in which

A_{C} is the cross-section area of the guy cable,

E_{eq} is the effective modulus of elasticity of the guy cable (accounting for the sag according to **4.2.3**(3) and **4.2.3**(4), if required in accordance with (3), (4)),

ℓ is the length of the cable,

α is the angle of the guy cable with respect to the horizontal.

(6) If both the sag and the mass of the guy cable are significant, the possibility of impulsive loading on the mast from the cable in the seismic design situation should be taken into account.

8.3 Materials

(1)P **7.3**(1)P applies.

(2)P **6.2.1**(2)P applies.

(3)P **6.2.1**(3)P applies.

(4) The requirements in EN 1998-1:2004, **6.2** apply.

8.4 Damage limitation state

(1) **5.5**(2) applies.

(2) A limit on the relative displacements between horizontal stiffening elements, computed in accordance with **4.9**(3), should be specified for the particular project for the damage limitation state, depending on the mast function.

ANNEX A (Informative)
LINEAR DYNAMIC ANALYSIS ACCOUNTING FOR ROTATIONAL COMPONENTS OF THE GROUND MOTION

(1) When the rotational components of the ground motion during the earthquake are taken into account, the seismic action may be represented by three elastic response spectra for the translational components and three elastic response spectra for the rotational components.

(2) The elastic response spectra for the two horizontal translational components (x and y axes) and for the vertical component (z axis) are those given in EN 1998-1:2004, **3.2.2.2** and **3.2.2.3**.

(3) The rotation response spectrum is defined in an analogous way to the response spectrum of the translational components, i.e. by considering the peak response to the rotational motion of a rotational single-degree-of- freedom oscillator, with natural period T and critical damping ratio ξ.

(4) R^{θ} denotes the ratio between the maximum moment in the oscillator spring and the rotational moment of inertia about its axis of rotation. The diagram of R^{θ} versus the natural period T, for given values of ξ, is the rotation response spectrum.

(5) When results of a specific investigation or of well-documented field measurements are not available, the rotational response spectra may be determined as:

$$R_{x}^{\theta}(T) = 1{,}7\pi S_{e}(T) / v_{s}T \tag{A.1}$$

$$R_{y}^{\theta}(T) = 1{,}7\pi S_{e}(T) / v_{s}T \tag{A.2}$$

$$R_{z}^{\theta}(T) = 2{,}0\pi S_{e}(T) / v_{s}T \tag{A.3}$$

where:

R^{θ}_{x}, R^{θ}_{y}, R^{θ}_{z} are the rotation response spectra around the x, y and z axes, in rad/s^2;

$S_e(T)$ is the elastic response spectra for the horizontal components at the site, in m/s^2;

T is the period in seconds.

v_s is the average S-wave velocity, in m/s, of the top 30 m of the ground profile.. The value corresponding to low amplitude vibrations, i.e., to shear deformations of the order of 10^{-6}, may be used.

(6) The quantity v_s is directly evaluated by field measurements, or through the laboratory measurement of the shear modulus of elasticity G, at low strain, and the soil density ρ, and inverting expression (3.1) in EN1998-5:2004, **3.2**(1):

$$v_s = \sqrt{G / \rho}$$

(7) In those cases where v_s is not evaluated by experimental measurements according to (6), the value from Table A.1 may be used, representative of the ground type of the site:

Table A.1: Default values of shear wave velocity for the five standard ground types

Ground type	Shear wave velocity v_s m/sec
A	800
B	580
C	270
D	150

(8) When a translational ground acceleration $\ddot{x}(t)$ is considered along horizontal direction x together with a rotation acceleration $\ddot{\theta}(t)$ in the vertical plane x-z, then, if the inertia matrix is [M], the stiffness matrix is [K], and the damping matrix is [C], the equations of motion for the resulting multi-degree-of-freedom system are given by:

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = -(\{m\}\,\ddot{x} + \{m\,h\}\,\ddot{\theta}) \qquad \text{(A.4)}$$

where:

$\{\ddot{u}\}$ is the vector comprising the accelerations of the degrees of freedom of the structure relative to the base;

$\{\dot{u}\}$ is the vector comprising the velocities of the degrees of freedom of the structure;

$\{u\}$ is the vector comprising the displacements of the degrees of freedom relative to the base;

$\{m\}$ is the vector comprising the translational masses in the horizontal direction of the translational excitation. This vector coincides with the main diagonal of the mass matrix [M], if the vector $\{u\}$ includes only the translational displacements in the horizontal direction of the excitation;

$\ddot{x}(t)$ is the translational ground acceleration, represented by S_e;

$\ddot{\theta}(t)$ is the rotational acceleration of the base, represented by R^{θ}.

(9) To account for the term $\{m\}$, the participation factor in the modal analysis of mode k is:

$$a_{ku} = \frac{\{\Phi^{T}\}\,\{m\}}{\{\Phi^{T}\}[M]\,\{\Phi\}} \qquad \text{(A.5)}$$

while, for the term $\{m\,h\}\,\ddot{\theta}$, the participation factor is:

$$a_{k\theta} = \frac{\{(\Phi\,h)^{T}\}\,\{m\}}{\{\Phi^{T}\}[M]\,\{\Phi\}} \qquad \text{(A.6)}$$

where:

$\{\Phi\}$ is the k-th modal vector

$\{\Phi h\}$ is the vector of the products of the modal amplitude Φ_i, at the i-th degree-of-freedom, and its elevation h_i .

(10) The effects of the two forcing functions should normally be superimposed in the time domain. They are generally not in phase, and accordingly the effects of the rotational ground excitation may be combined with those of the translational excitation via the SRSS (square root of the sum of the squares) rule.

ANNEX B (Informative)
MODAL DAMPING IN MODAL RESPONSE SPECTRUM ANALYSIS

(1) When the design response spectrum is applied, the value of the behaviour factor q incorporates energy dissipation in the elastic range of structural response, energy dissipation due to soil structure interaction, and energy dissipation due to the hysteretic behaviour of the structure. When the elastic spectrum is used in the analysis, the damping ratio (relative to the critical damping) needs to be explicitly defined. When a modal analysis is performed, the damping factors need be defined for each mode of vibration. If a mode involves essentially a single structural material, the damping ratio should conform to the dissipation properties of the material and should be consistent with the amplitude of deformation.

(2) For the most common structural materials, the damping values given in EN 1998-2:2005, **4.1.3** may be used

(3) If non-structural elements are considered to contribute to energy dissipation, higher values of damping may be assumed. Due to the dependency on the amplitude of deformation, in general lower bound values of the ratios are suitable for the damage limitation seismic action, while upper bound values of the ratios are suitable for the design seismic action. These bounds may be taken as:

- for ceramic cladding: 0,015–0,05;
- for brickwork liner: 0,03–0,10;
- for steel liner: 0,01–0,04;
- for fibre reinforced polymer liner: 0,015–0,03.

(3) Representative ranges of the damping ratio for the dashpots modelling energy dissipation in the soil, are:

- for the horizontal degree of freedom (swaying soil compliance): 0,10–0,20
- for the rotational degree of freedom (rocking soil compliance): 0,07–0,15
- for the vertical degree of freedom (vertical soil compliance): 0,15– 0,20

(4) Low damping ratios should be assigned to the dashpots of foundations on a shallow soil deposit underlain by bedrock or ground of similar stiffness.

(5) In general, for the type of structures addressed by this Eurocode, any mode of vibration involves the deformation of more than one material. In this case, for each mode, an average modal damping based on the elastic deformation energy stored in that mode is appropriate.

(7) The formulation leads to

$$\bar{\xi}_{\mathrm{j}} = \frac{\{\phi\}^{\mathrm{T}}\left[\overline{K}\right]\{\phi\}}{\{\phi\}^{\mathrm{T}}\left[K\right]\{\phi\}} \qquad \text{(B.1)}$$

where:

$\bar{\xi}_{\mathrm{j}}$ is the equivalent modal damping ratio of the j-th mode;

$[K]$ is the stiffness matrix;

$[\overline{K}]$ is the modified stiffness matrix, with terms equal to the product of the corresponding term of the stiffness matrix $[K]$, multiplied by the damping ratio appropriate for that element, and

$\{\phi\}$ is the j-th modal vector.

(8) Other techniques may also be used, if more detailed data on the damping characteristics of structural subsystems are available.

(9) It is recommended that the value of $\overline{\xi}_{\mathrm{j}}$ does not exceed 0,15, unless justified by experimental evidence.

ANNEX C (Informative)
SOIL-STRUCTURE INTERACTION

(1) This annex contains information supplementary to that of Informative Annex D of EN 1998-5:2004.

(2) The design earthquake motion is defined at the ground surface in free-field conditions, i.e. where it is not affected by the inertial forces due to the presence of structure. When the structure is founded on soil deposits or soft ground, the resulting motion at the base of the structure will differ from that at the same elevation in the free-field, due to the soil deformability. For tall structures, the rocking compliance of the soil may be important and may significantly increase the second order effects.

(3) The modelling methods of soil-structure interaction should take into account:

(a) the extent of embedment,

(b) the depth to the possible bedrock,

(c) the layering of the soil strata,

(d) the variability of the soil moduli in any single stratum, and

(e) the strain-dependence of soil properties (shear modulus and damping).

(4) The assumption of horizontal layering may generally be considered to apply.

(5) Unless the soil investigation suggests a suitable range of variability for the dynamic soil moduli, an upper bound of the soil stiffness may be obtained by multiplying the entire set of the best estimates of the moduli by 2, and a lower bound by multiplying the entire set by 0,5.

(6) Being strain-dependent, damping and shear moduli for each soil layer should be consistent with the effective shear strain intensity expected during the seismic action considered. An equivalent linear method is acceptable. In this case the analysis should be performed iteratively. In each iteration the analysis is linear, but the soil properties are adjusted from iteration to iteration until the calculated strains are compatible with the soil properties used in the analysis. The iterative procedure may be performed for the free-field soil deposit, disregarding the presence of the structure.

(7) The effective shear strain amplitudes in any one layer, to be used to evaluate the dynamic moduli and damping in equivalent linear methods, may be taken as

$$\gamma_{\mathrm{eff}} = 0{,}65\gamma_{\mathrm{max,t}} \tag{C.1}$$

where $\gamma_{\mathrm{max,t}}$ is the maximum value of the shear deformation in the soil layer in the free-field during the seismic action considered.

(8) If the finite elements modelling method for is used for the soil, the criteria for determining the location of the bottom boundary and the side boundary of the region modelled should be justified. In general, the forcing functions to simulate the earthquake motion are applied at these boundaries. In such cases, it is required to generate an excitation system acting at boundaries such that the response motion of the soil media at the surface free field is identical to the ground motion due to the seismic action considered. The procedures and theories for generation of such excitation system should be presented.

(9) If the half-space (lumped parameters) modelling method is used, the parameters used in the analysis for the soil deformability should account for the layering. The variability of soil moduli, and strain-dependent properties should also be taken into account.

(10) Any other modelling methods used for soil-structure interaction analysis should be clearly explained.

(11) The decision not to take into account soil-structure interaction in the analysis should be justified.

ANNEX D (Informative)
NUMBER OF DEGREES OF FREEDOM AND OF MODES OF VIBRATION

(1) A dynamic analysis (e.g. response spectrum or time-history method) is used when the use of the lateral force method is not considered justified.

(2) The analysis should:

- take into account the rocking and translation response of the foundation;
- include a sufficient number of masses and degrees of freedom, to determine the response of any structural element and plant equipment;
- include a sufficient number of modes to ensure participation of all significant modes;
- provide the maximum relative displacement between supports of equipment or machinery (for a chimney, the interaction between internal and external tubes);
- take into account significant effects, such as piping interactions, externally applied structural restraints, hydrodynamic loads (both mass and stiffness effects) and possible nonlinear behaviour;
- provide "floor response spectra", when the structure supports important light equipment or appendices.

(3) The effective modal mass, M_i, in mode i, mentioned in **4.3.3.2**(2), is defined as:

$$M_i=[\{\phi\}^T[M]\{i\}]^2/\{\phi\}^T[M]\{\phi\} \tag{D.1}$$

where:

$\{\phi\}$ is the i-th modal vector;

$\{i\}$ is a column vector, with terms equal to 1 or 0, which represents the displacement induced in the associated degree of freedom when its base is subjected to a unit displacement in the direction of the seismic action component considered.

(4) The criterion indicated in **4.3.3.2**(2) does not ensure the adequacy of the mass discretisation if light equipment or a structural appendix is concerned. In that case the above condition might be fulfilled, but the mathematical model of the structure could be inadequate to describe the response of the equipment or appendix. When the analysis of the equipment or appendix is necessary, a "floor response spectrum", applicable for the floor elevation where the equipment/appendix is supported, should be developed. This approach is also recommended when a portion of the structure needs to be analysed independently, for instance, an internal masonry flue of a chimney, supported on individual brackets of the structural shell.

ANNEX E (Informative)
MASONRY CHIMNEYS

E.1 Introduction

(1) A masonry chimney is a chimney constructed of masonry units and mortar, hereinafter referred to as masonry. Masonry chimneys should be constructed, anchored, supported and reinforced as required in this Annex.

E.2 Footings and foundations

(1) Foundations for masonry chimneys should be constructed of concrete or solid masonry at least 300 mm thick and should extend at least 150 mm beyond the face of the chimney or support wall on all sides. Footings should be founded on natural undisturbed ground or engineered fill below frost depth. In areas not subjected to freezing, footings should be at least 300 mm below the ground surface.

E.3 Behaviour factor

(1) The behaviour factor q should be taken as equal to 1,5, corresponding to low dissipative behaviour.

E.4 Minimum vertical reinforcement

(1) For chimneys with a horizontal dimension up to 1 m, a total of four 12 mm diameter continuous vertical bars anchored in the foundation should be placed in concrete between leaves of solid masonry or placed and grouted within the cells of hollow masonry units. Grout should be prevented from bonding with the flue liner, to avoid restricting its thermal expansion. For chimneys with a horizontal dimension greater than 1 m, two additional 12 mm diameter continuous vertical bars should be provided for each additional metre in horizontal dimension or fraction thereof.

E.5 Minimum horizontal reinforcement

(1) Vertical reinforcement should be enclosed within 6mm diameter ties, or other reinforcement of equivalent cross-sectional area, at a spacing of not more than 400 mm.

E.6 Minimum seismic anchorage

(1) A masonry chimney passing through the floors and roof of a building should be anchored at each level of floor or roof which is more than 2 m above the ground, except where constructed completely within the exterior walls. Two 5 mm by 25 mm steel straps should be embedded into the chimney over a minimum length of 300 mm. Straps should be anchored by hooks around the outer bars, and should extend by 150 mm beyond the bent at the hook. Each strap should be fastened to a minimum of four floor joists with two 12 mm bolts.

E.7 Cantilevering

(1) A masonry chimney should not project as a corbel from a wall or foundation by more than half of the chimney wall thickness. A masonry chimney should not project as a corbel from a wall or foundation that is less than 300 mm in thickness unless it projects equally on each side of the wall. As an exception, at the second storey of two-storey buildings, corbelling of chimneys outside the exterior walls may be equal to the

wall thickness. The projection of a single course should not exceed one-half of the height of the masonry unit, or one-third of its bed depth, whichever is less.

E.8 Changes in dimension

(1) The chimney wall or chimney flue liner should not change in size or shape within 150 mm above or below the level where the chimney passes through a floor or a roof, or their components.

E.9 Offsets

(1) Where a masonry chimney is constructed with a fireclay flue liner surrounded by one leaf of masonry, the maximum offset should be such that the centreline of the flue above the offset does not extend beyond the centre of the chimney wall below the offset. Where the chimney offset is supported by masonry below the offset in a manner for which the chimney has been designed, the maximum offset limitations do not apply.

E.10 Additional vertical loads

(1) Chimneys should not support vertical loads in addition to their own weight unless they are designed for them. Masonry chimneys may be constructed as part of the masonry walls or concrete walls of the building.

E.11 Wall thickness

(1) Masonry chimney walls should be constructed of solid masonry units, or hollow masonry units grouted solid with not less than 100 mm nominal thickness.

ANNEX F (Informative)
ELECTRICAL TRANSMISSION TOWERS

(1) The design of structures for electrical power transmission and distribution, and of substation wire supports is typically controlled by wind loads, often combined with ice loads or by unbalanced longitudinal wire loads. The seismic design situation generally does not control their design, except when it includes high ice loads. Earthquake performance of these structures has demonstrated that seismic loads can be resisted based on traditional electrical transmission, substation and distribution wire support structure loading. Heavy equipment, such as transformers in distribution structures, may result in significant seismic loadings and distress.

(2) Earthquake damage to electrical transmission, substation wire support or distribution structures is often due to large displacements of the foundations due to landslides, ground failure or liquefaction. Such occurrences normally lead to local structural failure or damage, without complete loss of the integrity and the function of the structure.

(3) The fundamental frequency of these types of structure typically ranges from 0.5 Hz to 6 Hz. Single-pole types of structure have fundamental mode frequencies in the 0.5 Hz to 1.5 Hz range. H-frame structures have fundamental mode frequencies in the 1 Hz to 3 Hz ranges, with the lower frequencies in the direction normal to the plane of the structure and the higher ones in-plane. Four-legged lattice structures have fundamental mode frequencies in the 2 Hz to 6 Hz range. Lattice tangent structures typically have lower frequencies in this range; angle and dead end structures have higher frequencies in the range. These frequency ranges can be used to determine whether earthquake loading is likely to control the structural design of the tower. If it is, then a more detailed evaluation of the structure vibration frequencies and mode shapes should be performed.